Te Puke
Western Bay of Plenty District Council
16-Oct-2015

Process Performance Review

Te Puke Wastewater Treatment Plant

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Client: Western Bay of Plenty District Council

Co No.: 125014

Prepared by

AECOM Consulting Services (NZ) Ltd
8 Mahuhu Crescent, Auckland 1010, PO Box 4241 Shortland St, Auckland 1140, New Zealand
T +64 9 967 9200 F +64 9 967 9201 www.aecom.com

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Prepared by Zhuo Chen

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1.0 Project Overview

The Western Bay of Plenty District Council (WBOPDC) currently has consent (Resource Consent Numbers 02 4891 and 02 4889) to operate the Te Puke Waste Water Treatment Plant (WWTP) and discharge wastewater into the Waiari stream.

WBOPDC wishes to renew the existing resource consent which is set to expire in November 2016. In addition, WBOPDC seeks to obtain new Bay of Plenty Regional Council (BOPRC) resource consents for a 35-year term, to meet the future needs for the Western Bay of Plenty district and in particular, the current expected 30 % population increase by 2045.

This report summaries the findings of the operational and performance compliance review of the current WWTP. AECOM undertook a site inspection of the WWTP and operational processes on 30 June 2015,

The objectives of this operational and compliance performance review include:

- Flow and load projections for the required consent life (35 years)
- Operational performance review of current process units
- Identification of potential upgrade requirements or options

2.0 Information Reviewed

In preparation of this report, AECOM has reviewed the following supporting documents provided by WBOPDC:

- Smart Growth WBOPDC Population and Household Projection 2013-2053 (as adopted by Finance and Risk Committee 4 July 2014.
- Historical plant data including influent analytical results (2013-2015), compliance monitoring results (1998-2015), and rainfall records (2009-2015).
- Current resource consent (024891) and associated original application report dated December 1996.
- General information on the Te Puke Wastewater Treatment Plant including relevant projects in Councils Draft Long Term Plan.
- Draft “Te Puke Wastewater Treatment Plant Capacity Assessment” (November 2014) by Harrison Grierson.
- As Built Plans of the Te Puke Wastewater Treatment Plant (Rev E, issued in Dec 2014).
- WBOPDC Water and Wastewater Operators Procedure Manual (April 2013).
- Te Puke Wastewater Treatment Plant Operations Manual (January 2015) prepared by Opus.
- Te Puke Wastewater Treatment Plant Review (February 2002) by Opus.
- Te Puke Wastewater Treatment Plant Upgrade to 12,000 PE, Preliminary Design Report (October 2005) by Waste Solutions Ltd.
- Te Puke Sewage Treatment Plant: Implications of Proposed Waiari Stream Water Abstraction (March 2007) by Duffill Watts and King Ltd.

Some other plant operation data were also obtained via direct personal communication with the plant operators.

3.0 Influent Volumes and Loads

Annual daily average wastewater flow into the WWTP has been relatively stable at around 1800 m^3/day over the past few years, with occasional spikes of peak flows reaching up to 3000 m^3/day as shown in Figure 1. The flow pattern shows a relatively weak correlation with rainfall records, without any obvious seasonal trend. The peak inflows rarely reached above 4000 m3/day, which indicated a very low observed peaking factor. The original design applied a peaking factor of 2.35, which we believe is adequate and has been applied in this study as well.

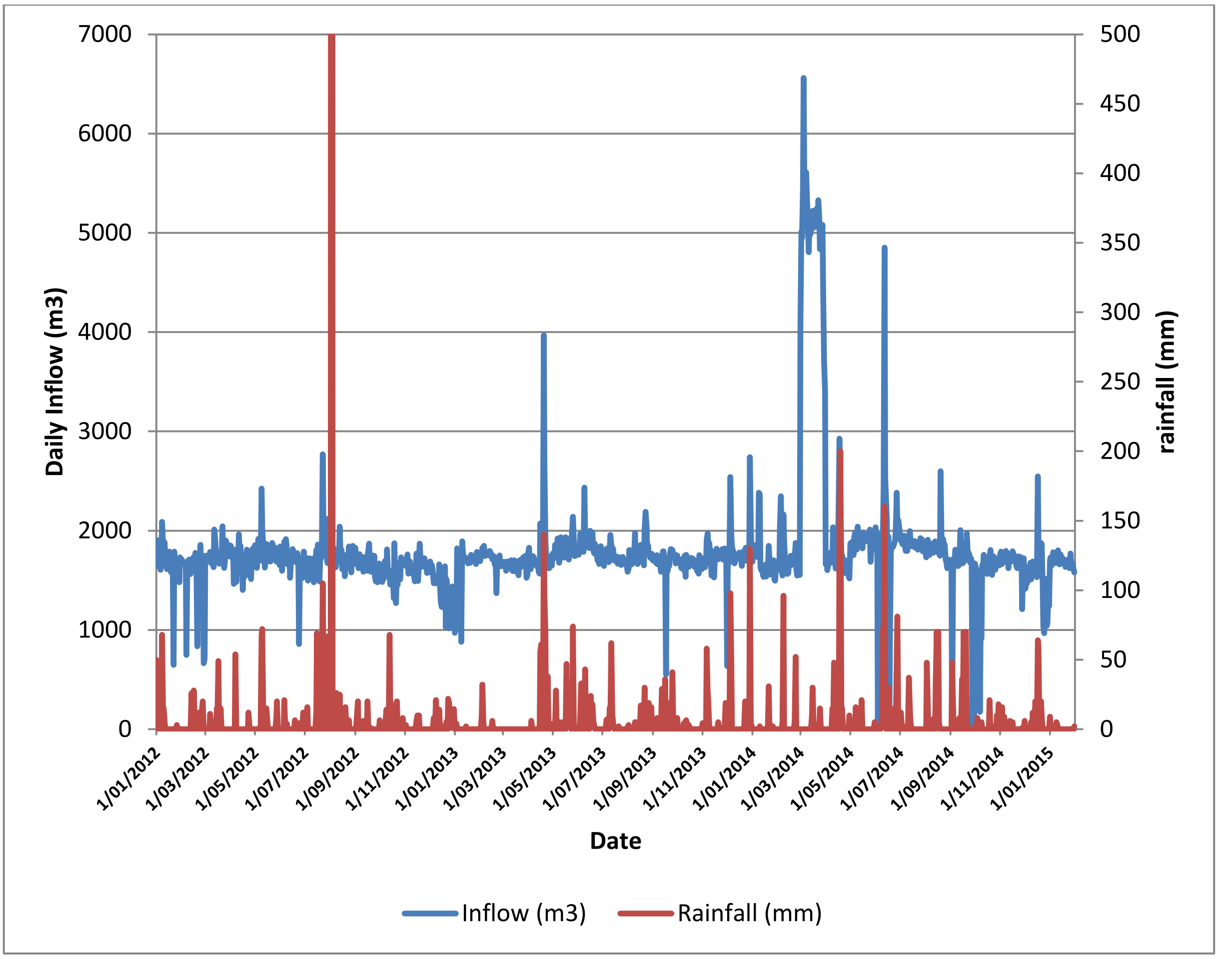

Figure 1 **Observed Daily Wastewater Inflow Volume into Te Puke WWTP and Rainfall Records.**

Based on the Smart Growth WBOPDC population projection, the current and projected connected populations are show in Table 1 below. The annual average flow (AAF) for the WWTP is calculated based on existing observed per capita flow of 223 L/day.

Table 1 **Projected population connection to Te Puke WWTP and expected wastewater flows**

Year	Population	Annual Average Flow (m3/day)	Peak wet weather flow (m3/day) (assuming a peaking factor of 2.35)
2015	8065	1800	4230
2016	8144	1818	4271
2021	8510	1899	4463
2031	9180	2049	4815
2041	9851	2199	5167
2051	10522	2348	5519

Various influent composition parameters have been monitored using S::Can probe, administered by DCM Process Control. The reported parameters include total suspended solids (TSS), Nitrate-nitrogen, COD, filterable COD, Ammoniacal nitrogen. This probe relies on spectrophotometry to correlate light absorbance/dispersion of the sample at particular wavelength with the measured wastewater constituents. Because a regular lab measurement regime and corresponding probe calibration is missing, AECOM does not consider that the reported influent analytical data are representative of the actual wastewater strength. In addition, some key wastewater characterisation parameters have not been monitored such as BOD, TKN, and total phosphorus, as such AECOM have not used this data for the review. In order to assess the capacity and process performance of the key process units, AECOM has applied typical wastewater characteristics observed in other areas within Bay of Plenty (i.e. Maketu) and derived the typical wastewater load based on the population and flow. This is summarised in Table 2.

Table 2 Typical wastewater load observed in other areas within Bay of Plenty

Parameters	Typical concentration (mg/L)	Current Load (kg/d)	Current load per day per capita (g/d/p)
BOD	375	675	84
COD	660	1188	147
TSS	320	576	71
Ammonia	60	108	13
TKN	75	135	17
Total Phosphorous	15	27	3

4.0 Current Process

Currently the wastewater from Te Puke reaches the WWTP into a wet well containing two submergible pumps working in a manual operation duty/standby mode. From the wet well the wastewater is pumped onto a 3-mm screen to remove large particles and other materials. The screenings are collected in bags and sent to landfill. From the screen, the wastewater is split into two secondary reactors via two Sutro weirs. The secondary reactors are composed of an initial anoxic zone and an aerobic zone with a mixed liquor recycling stream returning nitrified mixed liquor from the aeration zone into the anoxic zone.

Effluent from the secondary reactors flows by gravity to a splitter box, from where it is feed into three circular clarifiers. Return activated sludge (RAS) is directed into the after-screen chamber, while the wasted activated sludge (WAS) is pumped into two sludge holding tanks before being dewatered via centrifuge. Clarified effluent flows into a tertiary brush clarifier for further solid polishing before entering the UV plant for disinfection. The disinfected effluent is allowed to flow through a constructed up-flow wetland before being discharged into the Waiari Stream via a constructed riparian wetland.

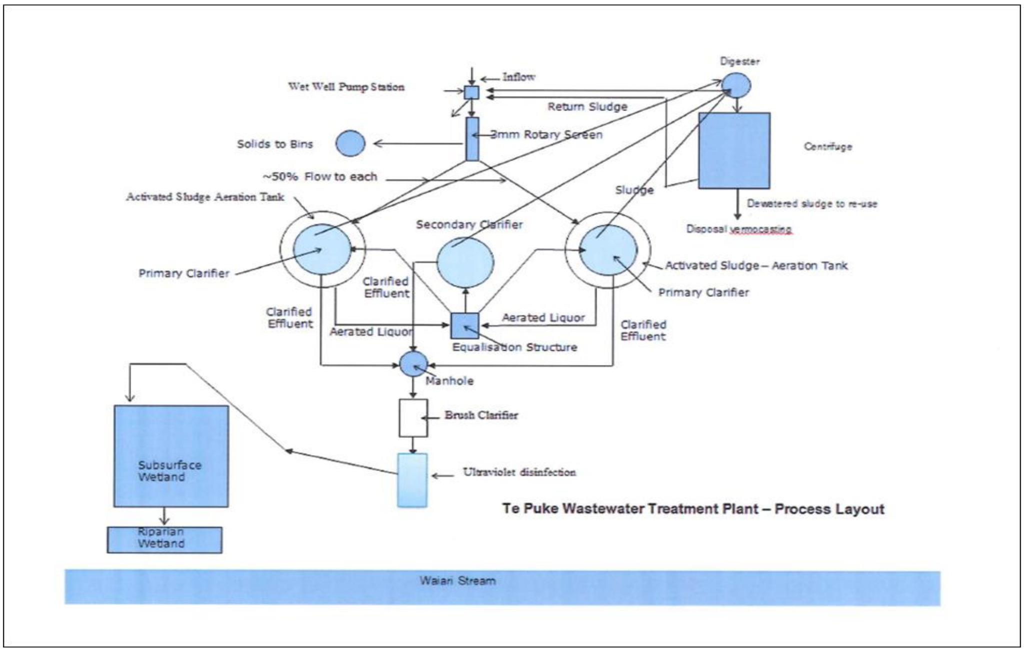

Figure 2 Te Puke WWTP Process Layout

5.0 Issues on Consent Compliance

Current resource consent (024891) includes the following effluent quality conditions:

Table 3 Current Te Puke WWTP Resource Consent Effluent Quality Limits

Parameter	Median	Maximum	Maximum Load
Flow		9000 m3/day	
cBOD5	-	30 g/m3	55 kg/day
TSS	-	30 g/m3	60 kg/day
TN	-	-	90 kg/day
DRP	-	20 g/m3	-
Faecal coliforms	200 /100 mL	1000 /100 mL	-

Monthly compliance monitoring data have been provided to AECOM for the purpose of this review. Although no comprehensive compliance assessment has been undertaken by AECOM, some key issues associated with the operating consent and compliance were identified, as presented in this section.

Generally good compliance with the consent limits has been achieved at Te Puke WWTP. This however may be due to the fact that the consent conditions, set in1998, were not as stringent as currently expected. For instance, the maximum flow level of 9000 m^3/day is not expected to be exceeded even in 2051. The maximum load of 90 kg/day of total nitrogen (TN) and maximum level of 20 mg/L dissolved reactive phosphorus (DRP) both appear to be relatively easy to achieve. Apart from these, no nutrient discharge limits have been included in the current consent. This shall be addressed in the suggested consent conditions proposed as part of the consent renewal process, refer to Table 4.

Apart from faecal coliforms, no median or other statistical levels for BOD, TSS, TN, DRP have been specified in the existing consent. AECOM is not aware of the development and subsequent negotiation of the current consent conditions. It's usually undesirable to apply maximum levels for treated wastewater effluent quality; even though maximum levels of 30 g/m^3 BOD and 30 g/m^3 of TSS are generally quite achievable with current practice within NZ. Occasional system failure can occur at any biological wastewater treatment plants, which may be caused by extreme flood events, surge of industrial wastewater flow, or microbial system breakdown, etc. For the consent renewal application, AECOM recommends the change of maximum levels to suitable median values or other statistical levels such as 92 percentile, etc.

Under the RMA framework and particularly the new operational National Policy Statement for Freshwater Management, it is expected that more stringent requirements on nutrients discharge are likely to be posed for this consent renewal. Due to the high dilution factors that can be achieved within Waiari Stream and Kaituna River (Refer to AECOM report: Te Puke WWTP – Public Health Risk Assessment), a TN level of 20 g/m^3 and TP level of 15 g/m^3 may be considered relevant for Te Puke WWTP effluent discharge. Judging by the level of DRP currently analysed monthly at the treatment effluent point, the TP level of 15 g/m^3 may be relatively achievable. Total nitrogen of 20g/m^3, however, is likely to be challenging for the plant. Current total nitrogen level observed in the treated effluent ranges from 8 to 30 g/m^3 with an average of approximately 20 g/m^3. This is largely due to the ineffectiveness of the current de-nitrification performance of the plant. In addition, the current consent does not include an ammonia or ammoniacal nitrogen discharge limit, which is often set at between 2-5 mg/L for other similar-sized plants. Observed ammonia-nitrogen level within the treated effluent (analysed monthly) can often be elevated up to more than 10 mg/L, due to inadequacy in nitrification as identified in the current plant operation. Detailed discussion on the plant process capacity and performance is presented in Section 6. It is therefore necessary for WBOPDC to undertake relevant plant upgrade and optimisation work to increase the nutrient reduction performance of the Te Puke WWTP.

The faecal coliform consent conditions currently specified in the consent are considered acceptable. However due to the better indication of human health risk by E. Coli, it is recommended to use E. Coli in place of faecal coliform to indicate the likely public health risks in the treated effluent. Similar as above, AECOM recommends change of the maximum level of 1000 /100 mL to a suitable statistical level such as a 92 percentile, etc.

The suggested effluent limits for the new consent conditions are summarised in table 4:

Table 4 AECOM suggested consent conditions in comparison with current conditions

	Current			Suggested	
Parameter	Median	Maximum	Maximum Load	10 out of 12 consecutive samples	Maximum
Flow		9000 m^3/day		4000 m^3/day	9000 m^3/day
cBOD5	-	30 g/m^3	55 kg/day	20 g/m^3	-
TSS	-	30 g/m^3	60 kg/day	30 g/m^3	-
TN	-	-	90 kg/day	20 g/m^3	-
DRP	-	20 g/m^3	-	-	-
TP	-	-	-	15 g/m^3	-
Faecal coliforms	200 /100 mL	1000 /100 mL	-	-	-
E. Coli	-	-	-	200 /100 mL	1000 / 100 mL

6.0 WWTP Process Design and Operational Issues

6.1 Unit Capacity and Operation Issues

6.1.1 Inlet Works

The current wet well is approximately 10-m in depth (personal communication) with two submergible pumps (Flygt 3127) working on a duty/standby mode. It is unknown to AECOM if variable speed drives have been added to the pump control. Based on the pump curve supplied by Flygt, a single pump will be adequate to cope with the expected annual average flow in 2051 and the relevant peak flow can be accommodated by two pumps.

Normally the wastewater flow is directed into the 3-mm Johnson Screen and Compactor which has a rated capacity of 70 L/s. This is equivalent to a daily flow of 6048 m^3/day. This is considered adequate for the projected annual average flow and peak wet weather flow up to 2051.

There is currently a standby step screen on site (Huber Step Screen L, dimensions unknown to AECOM). Although the slot width is unknown to AECOM, personal communication with the operator onsite during the site visit on 30th June 2015 revealed that, due to the age of the asset, the step screen can only retain particles or materials larger than 6 - 10 mm. This is therefore not a reliable standby unit for the operating 3-mm screen. Another 3-mm screen is required for the plant to provide adequate redundancy for inlet screening.

Grit removal is not provided in the WWTP, which may have negative impact on the performance of the secondary reactors. Grit accumulation in the aeration tank can clog mechanical equipment, reduce the aeration efficiency, and cause elevated maintenance costs. It is therefore recommended to include a grit removal unit downstream of the inlet screen before the wastewater flows into the secondary reactors.

6.1.2 Flow Spitting

Immediately post the screening, the wastewater is split into two streams which lead to two operational secondary reactors (reactor 1 and reactor 3). This is achieved by 2 Sutro weirs manually set to ensure that the two streams are proportionally sized to the respective reactor sizes. This is very inflexible operation-wise, causing issues on MLSS level control and sludge age control within the reactors. Operator often needs to manually adjust the weir level to control the flow into each reactor. It will be ideal to introduce an automatic level control component for the Sutro weirs, providing flexibility.

6.1.3 Secondary bioreactor/clarifiers

There are currently two secondary bioreactors with the working capacity of approximately 440 m^3 and 510 m^3, respectively. This provides a total volume of approximately 950 m^3 for aeration. At current annual average flow rate of 1800 m^3/day, this is equivalent of a hydraulic retention time of approximately 12 hr. According to the operation team, the current sludge retention time within the aeration tanks is approximately 4.5 days.

In absence of detailed inflow characteristics (refer comments section 3.0), AECOM has undertaken a high level design review of the aeration tank based on typical wastewater composition as specified in Table 2, current operation records, and other design assumptions such as biological growth kinetics applied elsewhere in New Zealand. A detailed summary of the calculation is shown in Appendix A. With the process modelling missing, this calculation provides a high level assessment of the sizing of the key equipment currently existent at the WWTP, including aeration tank, clarifier, blower, centrifuge, etc. Capacity constraints with various process units are also identified in the context of the projected inflow up to 2051.

A brief summary of the current plant operation is summarised below in Table 5.

Table 5 Summary of current plant operation parameters and AECOM recommended parameters

Parameters	Unit	Current Value	AECOM suggestion	Comments
AAF	m^3/d	1800	1800	Based on current inflow measurement record.
BOD	g/m^3	375	375	Based on typical wastewater characteristics experienced in New Zealand, particularly in Bay of Plenty region.
COD	g/m^3	660	660	Based on typical wastewater characteristics experienced in New Zealand, particularly in Bay of Plenty region.
TSS	g/m^3	320	320	Based on typical wastewater characteristics experienced in New Zealand, particularly in Bay of Plenty region.
Ammonia	g/m^3	60	60	Based on typical wastewater characteristics experienced in New Zealand, particularly in Bay of Plenty region.
TKN	g/m^3	75	75	Based on typical wastewater characteristics experienced in New Zealand, particularly in Bay of Plenty region.
SRT	days	4.5	9	Current SRT of 4.5 days was advised by plant operator. The recommended SRT of 9 days allow adequate time for nitrification and likely variation of peak influent TKN levels.
Liquid depth	m	4	4	Liquid depth of 4.0 metre for the aeration tanks was advised by plant operator on site.
Point of air release	m	0.5	0.5	Assuming that the air diffusers are located approximately 0.5 m above tank bottom.
D.O. in aeration basin	g/m^3	1.5	1.5	This is the current measured DO level within the aeration tanks. AECOM would recommend increasing this DO level up to at least 2.0 g/m^3. However this may be limited by the efficiency of the current air transfer system.
Selected MLSS concentration (MLSS)	g/m^3	2500	3000	Current MLSS concentration is maintained at approximately 2500 g/m^3. AECOM recommends a level of 3000 g/m^3.
Size of aeration tank required (V)	m^3	942.5	1397	Value calculated by AECOM based on assumptions as detailed in Appendix A. The calculated volume for existing system indicates that current aeration volume of 950 m^3 is adequate for BOD removal. However a significant increase in aeration volume is required to accommodate adequate nitrification.
aeration tank detention time (HRT)	hr	12.6	18.6	Value calculated by AECOM based on assumptions as detailed in Appendix A. An increase in HRT is recommended to allow adequate nitrification.
Oxygen demand R_0	kg/d	612	1119	Value calculated by AECOM based on assumptions as detailed in Appendix A.
	kg/h	25.5	46.64	
Air flow requirement	m^3/min	13	18.4	Value calculated by AECOM based on assumptions as detailed in Appendix A.
	m^3/h	783	1102	

Parameters	Unit	Current Value	AECOM suggestion	Comments
TSS in RAS (Xr)	g/m^3	9000	9000	Value supplied by WBOPDC Operation Team. This may not be representative of actual TSS level within the RAS flow.
Recycling ratio R		0.38	0.5	Value calculated by AECOM based on assumptions as detailed in Appendix A. Current RAS recycling rate is approximately 570 m^3/d, equivalent to a recycling ratio of 0.32. This is relatively close to the calculated value of 0.38.
Total clarifier area required	m^2	81.8	81.8	Using 22 $m^3/(m^2.d)$ hydraulic loading rate. This, however, may be optimistic, pending confirmation of the SVI value of the sludge.
Each clarifier area required	m^2	27.3	27.3	Assuming 3 clarifiers in operation.
Clarifier diameter required	m	5.89	5.89	Assuming 3 clarifiers in operation. If one clarifier is taken out of service for maintenance, the required clarifier diameter is calculated to be 7.2 meters, still below the existing clarifier diameters.
Clarifier solid loading rate	kg MLSS $/m^2.h$	3.33	4.4	This assumes all 3 clarifiers in operation. The solid loading rate is at a lower end of an acceptable range.
WAS flow (Qw)	m^3/d	58	52	Value calculated by AECOM based on assumptions as detailed in Appendix A. AECOM suggested WAS flow is reduced due to the increase of sludge retention time and MLSS level within the aeration tanks. This may benefit the operation and management of downstream sludge dewatering and disposal steps.
	m^3/h	2.4	2.2	
WAS dry solids (TSSw)	kg/d	523.6	466	Value calculated by AECOM based on assumptions as detailed in Appendix A. Reduction of WAS dry solid load can be expected when sludge retention time and MLSS level are increased.
WAS wet solid	ton/month	99.5	88.6	Assuming dewatered sludge cake contains 16% dry solids. The value of 99 ton/month appears slightly higher than WBOPDC's operation record of approximately 80 tonne/month. This is likely caused by the unknown WAS TSS level, and varied MLSS level within the reactors.

It appears that with a sludge retention time of 4.5 days, the current aeration basin size (950 m^3) is considered adequate for BOD removal. The clarifiers are also adequately sized in terms of hydraulic loading rate and solid loading rate. However, the aeration basin is at its capacity. Any further increase of inflow will require increase of MLSS concentration within the aeration basin or upsizing of the aeration basin itself.

Current sludge retention time of 4.5 days is not considered adequate for nitrification. This is particularly the case when the typical influent ammonia levels and TKN levels are high in New Zealand. This has been pointed out by the recent capacity assessment work (Harrison Grierson 2014). Based on a widely-acceptable growth rate of nitrifying bacteria, the sludge retention time for nitrification should be no less than 6 days. If a safety factor of 1.5 is considered (i.e. ratio of peak TKN over average TKN), the recommended sludge retention time to allow both BOD removal and nitrification should be approximately 9 days. In addition, AECOM considers that the current MLSS concentration of 2500 mg/L is too low, which may cause excessive sludge wasting and inadequate BOD removal. AECOM understands that the operating MLSS level is possibly limited by the aeration capacity and downstream WAS processing capacity. However a MLSS concentration of 3000 mg/L should be maintained within the aeration basin for adequate BOD reduction and nitrification. The modified calculations applying a 9-day SRT and 3000 mg/L MLSS are also presented in Table 5 above under the “AECOM suggestion” column.

It can be seen that current aeration tanks are already under capacity, if full nitrification is to be accommodated. This issue may be attenuated by further increasing the MLSS level, which would mandate higher RAS recycling rate and higher solid loading rate on the secondary clarifiers. Current RAS recycling rate is set at approximately 0.32.

The calculated air flow requirement for both BOD removal and nitrification is found to be approximately 1102 m^3/hr at current annual average flow of 1800 m^3/day. This, in theory, is achievable with the current Aerzen

Positive Displacement blower GM 50L, which has a maximum air volumetric flow of 2700 m^3/hr (for DN150). This is also achievable with the other blower installed on site (EasyAir EAX2 200 RAMX 400), which as an inlet volume range of approximately 1800 -2500 m^3/hr, depending on the pressure. However, it shall be noted that the air transfer efficiency of the existing diffusers is unknown to AECOM. According to the operation team the current coarse bubble diffusers were installed in the 1970s.. This is reflected by the low DO level (approximately 1.5 mg/L) that can be maintained within the aeration tank. AECOM understand all diffusers and sparge pipes were lifted and serviced as part of the 2010/2011 operational maintenance works.

The volume of the anoxic chamber currently included upstream of the aeration basin is unknown to AECOM. The design basis of this anoxic chamber is unclear as well. These include: target level of denitrification, sizing calculation, design consideration of alkalinity, oxygen credit, and food/microorganisms ratio, etc. A detailed modelling or assessment of the denitrification capacity is beyond the scope of this study. Provided that the influent TKN level is relatively high, it appears that the current anoxic zone is too small to achieve any significant denitrification. For instance, a moderate hydraulic retention time of 2.5 hr within the anoxic zone will require a volume of 187 m^3 with the current average flow rate of 1800 m^3/day. This will be equivalent to approximately 1/5 of the total secondary reactor volume.

The key capacity and operational issues with the current secondary reactor/clarifier can be summarised as following:

- The aeration efficiency is low, possible due to the aging air diffuser system and the likely accumulation of grits within the aeration tank. AECOM understand the aeration tanks were drained and grit removed as part of the 2010/2011 operational maintenance works.
- The current sludge retention time (SRT) is inadequate to facilitate required nitrification. A slight increase of MLSS concentration and an increase of SRT to 9 days are recommended. This would require a RAS recycling rate of 0.5, assuming that the suspended solid concentration with the sludge wasting or recycling line is approximately 9000 mg/L.
- Denitrification as facilitated by the existing anoxic zone upstream of the aeration basin is considered inadequate. This is reflected in the relatively high nitrate level in the treated effluent.
- Apart from other operational issues, the size of the current aeration tanks is at capacity. Any further increase of influent or accommodation of nitrification requirement will require the upgrade of the aeration tanks. For an annual average flow of 2348 m^3/day (i.e. 2051 design horizon), the total required size for the aeration tanks would be approximately 1822 m^3 if an MLSS level of 3000 mg/L is to be maintained. This is equivalent to almost doubling the existing aeration tank size.
- In theory, the current clarifiers are considered adequate regarding both hydraulic loading rate and solid loading rate for both current and future (2051) scenarios. This, however, is based on good settling characteristics of the activated sludge, which can be problematic when bulking filamentous sludge is developed.

6.1.4 Sludge dewatering

Wasted activated sludge (WAS) from clarifiers is drawn into two sludge holding tanks (with working volume of 27 m3 and 30 m3, respectively), before dosed with polymer and loaded onto centrifuge for dewatering.

The centrifuge currently installed on site (installed in 2005) is an Alfa Laval Decanter Centrifuge (Model Aldec 406). The design loading rate as advised by WBOPDC is 10 m3/hr with a solid capacity of 100 kg DS/hr. As illustrated in the calculation sheet attached in Appendix A, the WAS production flow currently observed at the WWTP should be approximately 57 m3/day. Hence the hydraulic retention time within the sludge holding tanks is approximately 23.5 hrs, close to targeted 24 hr retention time. It shall be noted that this calculation was based on a TSS level of 9000 mg/L within the WAS, which was provided by WBOPDC. From previous observation (Harrison Grierson 2014), the WAS solid content was reported to be only 6000 mg/L, which would result in a WAS flow of 87 m3/day and a retention time of only 15 hr within the sludge holding tanks. This is probably more in line with actual operation record. During AECOM's site visit on 30th June 2015, the plant operator advised that the current hydraulic retention time for the sludge holding tanks is approximately 12-15 hrs. Higher SRT and higher MLSS concentration within the aeration tank will result in a reduction of WAS production (approximately 77 m3/day if a TSS concentration of 6000 mg/L within WAS is applied). This will allow a combined retention time for the sludge holding tanks to be slightly higher than 17 hours. Increase of influent to 2348 m3/day (AAF for 2051) will increase the WAS production rate to approximately 101 m3/day and reduce the storage retention time to 13 hours. Although no capacity issues have been identified for the centrifuge, the flexibility of the sludge disposal is limited due to the lack of centrifuge redundancy and relatively small holding capacity within the sludge holding tanks. Currently the dewatered sludge cake contains approximately 12-16% dry solids (personal communication with WBOPDC Operation Team). This can be improved by thickening or pre-consolidation of the sludge before centrifuge.

6.1.5 Tertiary brush filter

All secondary clarifier effluent is blended and allowed to go through a brush filter composed of two identical modules. Each module can be lifted manually with a davit hoist installed on site. The design basis of the tertiary brush clarifier is unknown to AECOM.

On general terms, the clarifier effluent should meet the UV design criteria regarding the UV transmittance or TSS concentration. A tertiary filter will provide further insurance on the performance of the subsequent UV disinfection. However, no TSS monitoring data prior and post the tertiary brush filter are available for assessment of its actual performance. The solid removal in the brush is visible during the site visit. According to the plant operator, the main issue of the filter operation is the passive design of the brush filter requiring regular lifting and cleaning of the brushes on a once every fortnight basis.

6.1.6 UV disinfection

Effluent from the tertiary brush filter is disinfected by a Trojan UV3000Plus unit, with a peak design flow of 5417m^3/day. This is close to the projected peak wet weather flow for 2051. Hence no capacity issue with the UV disinfection unit is considered.

6.1.7 Wetland

A summary of the performance assessment of the vertical flow wetland was provided in the Te Puke Wastewater Treatment Plant Review by Opus (2002). This review was completed after the flow-tracing study undertaken by NIWA in 2001 and highlighted a few fundamental flaws associated with the design of this wetland. These included the tendency of pipe blockage or clogging of the gravel media and the likelihood of very uneven effluent distribution through the basal pipe network. This has been confirmed by the plant operator who advised that the wetland has been suffering from uneven distribution of the effluent since commissioning and currently only 1 out of constructed 5 effluent outlets is still functional.

6.2 Summary and Recommendations

In summary, there are a number of operation and capacity issues identified for Te Puke WWTP. This is based on a very high level assessment of the existing process units and operation. A brief list of the findings associated with each process unit is provided blow in Table 6. The proposed upgrades however have not been prioritised.

Table 6 Summary of Unit Capacity and Potential Upgrade Requirements

Process Unit	Capacity or Operation Issues Identified	Proposed Upgrade
Inlet Works	- No issues identified for the two submersible pumps. - Step screen does not meet the operation requirements. - Redundancy for the 3-mm screen is not provided at present, resulting in inadequate screening when the screen is taken out of service for maintenance. - Grit removal is not in place, resulting in grits accumulation within the aeration tanks. - Sutro weirs inflexible for flow split adjustment.	- Provide full redundancy for the 3-mm screen. - Additional of a grit removal unit. - Provide automatic control of the Sutro weirs to allow easy adjustment of the flow split between the two secondary reactors.
Secondary reactors	- Difference sizes of the secondary bioreactors and the lack of easy control of the inflow splitting can cause issues in managing MLSS level and sludge age within the aeration tank. - Current sludge retention time is not adequate for nitrification. - Current MLSS level within the aeration tanks is relatively low. - Upgrade of the aeration tanks will be required to	- The air diffusion system is recommended to be upgraded to a fine bubble diffusor system, to increase the oxygen transfer rate. - The dissolved oxygen level within the aeration tank is recommended to be maintained at 2 mg/L, if possible. - The MLSS level is recommended

Process Unit	Capacity or Operation Issues Identified	Proposed Upgrade
	accommodate nitrification and future inflow increase. - Current oxygen level within the aeration tanks is low. - Aeration efficiency is an issue, due to the aging coarse bubble diffuser system and grits accumulation within the aeration tanks. - Based on a very high level evaluation as shown in Appendix A (without detailed design calculation or actual air measurement), the current blowers (Aerzen Positive Displacement GM 50L and EasyAir EAX2 200 RAMX 400) are considered adequate to deliver the required oxygen demand. However some mechanical issues regarding the blower operation have been raised by the plant operator. - Existing anoxic zone is considered inadequate for denitrification.	to be increased to a minimum 3000 mg/L. - The sludge retention time within the aeration tank is recommended to be increased up to 9 days to allow adequate nitrification. - It has been recommended previously to use the empty space within the previous blower room (underneath the current aeration tanks) to increase the size of the aeration tanks. Nevertheless the actual sizing of the upgraded aeration tanks cannot be determined at this stage without a detailed process model. Nevertheless, construction of a third aeration tank around the Clarifier 3 is likely to be needed in the near future. - Detailed analysis is required to determine the size of the anoxic zone and amount of mixed liquor recycled for the purpose of denitrification.
Clarifiers	- Based on a high level evaluation, the current clarifiers are considered adequate to meet the capacity requirement for the design horizon 2051. - However this high level evaluation is based on a standard hydraulic loading rate for the secondary clarifier, assuming satisfactory settlability of the sludge. This is to be confirmed by actual measurement and detailed process modelling. - No significant issues were raised by plant operator regarding the flow splitting at the common secondary effluent chamber. However, asymmetric hydraulic and solid loading to the 3 reactors may occur frequently, which may lead to solid imbalance in various reactors and clarifiers.	- Continuous monitoring of the sludge SVI is recommended to ensure that the clarifiers are not overloaded. - Flow splitting among the three clarifiers may require attention to ensure that the flow is split correctly according to the operating volume of each clarifier.
Sludge Dewatering	- Sludge holding tanks' capacity is limited at present, which demands frequent operation of the centrifuge. - The centrifuge capacity is considered adequate, but no redundancy is provided at the moment, which may cause issues when the centrifuge is taken out of service. - Current centrifuge feed contains a low dry solid content (approximately 0.6-0.8%), which results in the low dry solid content of the dewatered sludge cake (approximately 16-17%). - Centrifuge dewatered sludge cakes are currently collected by trucks sitting underneath the centrifuge discharge chute, which is not an efficient use of the trucks.	- It is recommended to increase the capacity of the sludge holding tanks to provide flexibility on centrifuge operation. - AECOM recommends the installation of a second centrifuge to provide full redundancy of the centrifuge. - AECOM recommends the addition of a pre-thickening or other solid consolidation step prior to the centrifuge. This may involve installing a gravity thickener or other similar unit to

Process Unit	Capacity or Operation Issues Identified	Proposed Upgrade
	- The unavailability of trucks to remove sludge reduces the amount of wasting.	produce a sludge dry solid content of up to 4% before centrifuge and returning the supernatant to the inlet works. - AECOM recommends the usage of sludge cake collection bins, which can be covered, stored on site, and emptied periodically by trucks when full.
Tertiary brush filter	- The performance of the brush filter in terms of TSS removal from the clarifier effluent is questionable. - Frequent maintenance of the filter modules poses an operation burden on the plant operation team.	- It is recommended to undertake a study investigating the TSS removal performance of the brush filter under various flow scenarios. - If the study shows that clarifier effluent meets the consent requirements on TSS and design specifications for the UV disinfection, the brush filter may be decommissioned or bypassed. - If further TSS removal is required due to more stringent consent conditions, it is recommended to install an automated tertiary filter.
UV Disinfection	- No capacity or performance issues have been identified with the current UV system.	- If required in the future, additional UV lamp module can be installed in the existing channel.
Wetland	- The design of the up-flow wetland has high tendency of pipe blockage or clogging of the gravel media. This leads to uneven effluent distribution through the basal pipe network underneath the constructed wetland and uneven effluent discharge into Waiari Stream. - Consequently, the current up-flow wetland provides no value in polishing the treated effluent. - In addition the wetland is prone to flood hazard. A detailed flooding hazard assessment is not available at this stage. However the plant operator has recalled a few storm events when the whole wetland was inundated.	- It may be applicable to reconfigure the wetland to a traditional horizontal flow wetland, which may induce a more even distribution of the treated effluent in the wetland. However a feasibility study will need to be completed to determine if additional wetland polishing step is required and if this can be achieved by the wetland. - In the short term, it may be applicable that the treated effluent be discharged directly after the UV disinfection via a re-constructed bank-side perforated diffuser pipe and a rock passage located along the river bank. This rock passage can be built along the existing gabion basket wall which appears to have minimal visual impact on the surrounding area.

7.0 Current CAPEX Programme

AECOM understands that there are a suite of projects that are currently included in the draft Long Term Plan (LTP) for Te Puke wastewater treatment plant. These include the upgrade work in the following areas:

- Fixed generator
- Sludge thickener
- Inlet screen
- Inlet grit removal
- Effluent monitoring equipment
- WAS holding tank
- Selector tank
- Tertiary filter upgrade
- Sludge building modifications
- Renewal works
    - Blowers
    - Transformer
    - Switchboard

It appears that the planned projects have been developed to meet the identified upgrade requirements identified earlier such as inlet works and sludge handling. The concept and development timeframe for the capacity upgrade of the secondary reactors are not clear to AECOM at this stage. Based on our high level assessment, the current aeration tanks are in need of upgrade to accommodate inflow increase and meet the likely more stringent consent conditions in the future.

The proposed upgrades identified in Table 6 will need to be reviewed against the suggested consent conditions proposed in the Consent Compliance, refer AECOM suggested consent conditions in comparison with current conditions Table 4 of this report, and WBOPDC planning strategy for the next LTP cycle. Projects can then be prioritised and a draft CAPEX programme developed for the 2018 to 2028 LTP.

8.0 Standard Limitation

AECOM Consulting Services (NZ) Limited (AECOM) has prepared this report in accordance with the usual care and thoroughness of the consulting profession for the use of Western Bay of Plenty District Council and only those third parties who have been authorised in writing by AECOM to rely on this Report.

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Any estimates of potential costs which have been provided are presented as estimates only as at the date of the Report. Any cost estimates that have been provided may therefore vary from actual costs at the time of expenditure.

Appendix A

Activated Sludge System Calculation

Te Puke WWTP Capacity Assessment **AECOM**
Scenario 1: Current System_BOD Removal Only 17/08/2015

	Value	flow	Load	Load per Capita
	g/m3	m3/d	kg/d	g/p/d
BOD	375	1800	675	84
COD	660	1800	1188	147
TSS	320	1800	576	71
Ammonia	60	1800	108	13
TKN	75	1800	135	17
Total Phosphorous	10	1800	18	2
sBOD	168.75	g/m3	Assuming sBOD/BOD = 0.45	
sCOD	290.4	g/m3	Assuming sCOD/COD = 0.44	
rbCOD	174.24	g/m3	Assuming rbCOD/sCOD = 0.6	
VSS	272	g/m3	Assuming VSS/TSS = 0.85	
y	0.4	g VSS/g bCOD	biomass yield	
Ks	20	g/m3	half velocity constant	
μm	6	g/g.d	Maximum specific growth rate at 20 C	
μm,12 C	3.49	g/g.d	Maximum specific growth rate at 12 C	
kd	0.09		endogenous decay coefficient	
S0	600	g/m3	influent concentration	
SRT	4.5	days		
FS	1.5		Assume TKN peak/TKN average (safety factor)	
S	1.9477717	g bCOD/m3	concentration of growth-limiting substrate in solution	
fd	0.15		fraction of cell mass remaining as cell debris	
Yn	0.12	g VSS/g Nox		
k dn	0.06	g/g.d		
Liquid depth	4	m		
Point of air release	0.5	m	air diffuser 0.5 m above tank bottom.	
α	0.5		aeration factor	
β	0.95		aeration factor	
F	0.9		diffuser fouling factor	
D.O. in aeration basin	1.5	g/m3		
bubble air transfer efficiency	0.35		Assumption	
bCOD	600	g/m3	biodegradable COD	
nbCOD	60	g/m3	non-biodegradable COD	
sCODe	20.4	g/m3	Effluent sCODe	
bpCOD	330	g/m3	biodegradable particulate COD	
pCOD	369.6	g/m3	particulate COD	
bpCOD/pCOD	0.8928571			
nbVSS	29.142857	g/m3	nonbiodegradable volatile solids	
iTSS	48	g/m3	inert total suspended solids	
Design for BOD removal only				
Px, vss	327.04133	kg VSS/d	biomass production	
The mass of VSS and TSS in aeration basin				
Px, vss	379.49847	kg/d		
Px, tss	523.61165	kg/d		
	1707.7431	kg	mass of MLVSS in Aeration basin	
	2356.2524	kg	mass of MLSS in Aeration basin	
MLSS	2500	g/m3	Selected MLSS concetration	
V	942.50097	m3	Size of aeration tank	
HRT	12.56668	hr	aeration tank detention time	
VSS/TSS	0.7247709		VSS fraction	
MLVSS	1811.9272	g/m3		
F/M and BOD volumetric loading				
F/M	0.3952585	kg/kg.d		
BOD loading	0.7161796	kg/m3.d		
Observed yield based on TSS and VSS				
bCOD removed	1076.494	kg/d		

Y obs, TSS	0.4864046	g TSS/g bCOD		
	0.7782474	g TSS/g BOD		
Y obs, VSS	0.3525319	g VSS/g bCOD		
	0.564051	g VSS/g BOD		
O2 demand				
R0	612.09532	kg/d		
	25.503972	kg/h		
Aeration design				
C (12 C)	10.77	mg/L	average dissolved oxygen satution concentration	
P	10.366	metre of water	Atmospheric pressure	
C (S,T,H)	12.075347	mg/L		
SOTR	74.002555	kg/h	standard oxygen transfer rate	
	13.051597	m3/min	Air flow	
	783.09582	m3/hr		
Secondary Clarifier Design				
Xr	9000	g/m3	Assumption	
R	0.3846154			
Area required	81.818182	m2	Using 22 m3/m2.d hydraulic loading rate	
Each clairifer	27.272727	m2		
Dimater	5.8942623	m2		
Solid loading	3.1730769	kg MLSS/m2.h		
Sludge Wasting				
Qw	58.179072	m3/d		
	2.424128	m3/h		
V sludge	57	m3	Sludge holding tank size	
T	23.51361	hr	Retention time	
TSSw	523.61165	kg/d		
	21.817152	kg/h		
	3.6652815	ton/week		
	15.926521	ton/month		

Te Puke WWTP Capacity Assessment **AECOM**

Scenario 2: AECOM suggested system_BOD Removal and Nitrification 17/08/2015

	Value	flow	Load	Load per Capita
	g/m3	m3/d	kg/d	g/p/d
BOD	375	1800	675	84
COD	660	1800	1188	147
TSS	320	1800	576	71
Ammonia	60	1800	108	13
TKN	75	1800	135	17
Total Phosphorous	10	1800	18	2
μn	0.16667	g/g.d	Nitrifying bacter growth rate	
sBOD	168.75	g/m3	Assuming sBOD/BOD = 0.45	
sCOD	290.4	g/m3	Assuming sCOD/COD = 0.44	
rbCOD	174.24	g/m3	Assuming rbCOD/sCOD = 0.6	
VSS	272	g/m3	Assuming VSS/TSS = 0.85	
y	0.4	g VSS/g bCOD	biomass yield	
Ks	20	g/m3	half velocity constant	
μm	6	g/g.d	Maximum specific growth rate at 20 C	
μm,12 C	3.49	g/g.d	Maximum specific growth rate at 12 C	
kd	0.09		endogenous decay coefficient	
S0	600	g/m3	influent concentration	
Desgin SRT	9	days		
FS	1.5		Assume TKN peak/TKN average (safety factor)	
SRT for Nitrification	6	days		
S	1.20728	g bCOD/m3	concentration of growth-limiting substrate in solution	
fd	0.15		fraction of cell mass remaining as cell debris	
Yn	0.12	g VSS/g Nox		
k dn	0.06	g/g.d		
Nox	60	g/m3	assuming NOX = 80% of TKN	
Liquid depth	4	m		
Point of air release	0.5	m	air diffuser 0.5 m above tank bottom.	
α	0.65		aeration factor	
β	0.95		aeration factor	
F	0.9		diffuser fouling factor	
D.O. in aeration basin	1.5	g/m3		
bubble air transfer efficiency	0.35		Assumption	
bCOD	600	g/m3	biodegradable COD	
nbCOD	60	g/m3	non-biodegradable COD	
sCODe	20.4	g/m3	Effluent sCODe	
bpCOD	330	g/m3	biodegradable particulate COD	
pCOD	369.6	g/m3	particulate COD	
bpCOD/pCOD	0.89286	g/m3		
nbVSS	29.1429	g/m3	nonbiodegradable volatile solids	
iTSS	48	g/m3	inert total suspended solids	
Design for BOD removal and nitrification				
Px, vss	277.91	kg VSS/d	biomass production	
NOx	55.9727	g/m3	Nitrogen oxidized	
The mass of VSS and TSS in aeration basin				
Px, vss	330.367	kg/d		
Px, tss	465.81	kg/d		
	2973.3	kg	mass of MLVSS in Aeration basin	
	4192.29	kg	mass of MLSS in Aeration basin	
MLSS	3000	g/m3	Selected MLSS concetration	
V	1397.43	m3	Size of aeration tank	
HRT	18.6324	hr	aeration tank detention time	
VSS/TSS	0.70923		VSS fraction	
MLVSS	2127.69	g/m3		
F/M and BOD volumetric loading				

F/M	0.22702	kg/kg.d		
BOD loading	0.48303	kg/m3.d		
Observed yield based on TSS and VSS				
bCOD removed	1077.83	kg/d		
Y obs, TSS	0.43218	g TSS/g bCOD		
	0.69148	g TSS/g BOD		
Y obs, VSS	0.30651	g VSS/g bCOD		
	0.49042	g VSS/g BOD		
O2 demand				
R0	1119.45	kg/d		
	46.6436	kg/h		
Aaeration design				
C (12 C)	10.77	mg/L	average dissolved oxygen satution concentration	
P	10.366	metre of water	Atmospheric pressure	
C (S,T,H)	12.0753	mg/L		
SOTR	104.109	kg/h	standard oxygen transfer rate	
	18.3613	m3/min	Air flow	
	1101.68	m3/hr		
Secondary Clarifier Design				
Xr	6000	g/m3	Assumption	
R	1			
Area required	81.8182	m2	Using 22 m3/m2.d hydraulic loading rate	
Each clairifer	27.2727	m2		
Dimater	5.89426	m		
Solid loading	5.5	kg MLSS/m2.h		
Sludge Wasting				
Qw	77.635	m3/d		
	3.23479	m3/hr		
V sludge	57	m3	Sludge holding tank size	
T	17.6209	hr	Retention time	
TSSw	465.81	kg/d		
	19.4087	kg/h		
	3.26067	ton/week		
	14.1684	ton/month		